REPORT ON:
147-151 NAPIER ROAD, HAVELOCK NORTH,

PROJECT:
GEOTECHNICAL INVESTIGATION

CLIENT: SUN PROPERTIES LTD

R_220833

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1 OVERVIEW

Sun Properties Ltd (LDL) engaged Resource Development Consultants Ltd (RDCL) to undertake a geotechnical investigation and assessment for the proposed residential subdivision at 147 – 151 Napier Road, Havelock North.

1.1 UNDERSTANDING THE PROJECT

Plans provided by the client indicate a residential development consisting of 22 Lots of various sizes are proposed to replace the existing buildings at 147 – 151 Napier Road.

A geotechnical assessment is required to support the application for resource consent.

1.2 RELEVANT GUIDELINES

Geotechnical investigations and assessment have been undertaken in accordance with:

- Resource Management Act (1992).
- Hastings District Council (June 2019); Geotechnical Site Investigation Guidelines for Residential Building Consents.
- MBIE (2016) Module 2 Geotechnical Investigations for Earthquake Engineering.
- Ministry of Business, Innovation and Employment (MBIE) guidelines, revised issue of Repairing and Rebuilding Houses Affected by the Canterbury Earthquakes. Part A: Technical Guidance (TC2 and TC3) version 3 (December 2012).
- MBIE guidelines, Part C: Assessing, Repairing and Rebuilding foundations in TC3, version 3a (April 2015).

1.3 SCOPE OF WORK

Work was undertaken in general accordance with RDCL proposal 220833.

2 SITE DESCRIPTION

The proposed development is located on a relatively flat ground, bordered by:

- Napier Road on the east;
- Residential developments on the north and south; and
- Karamu stream and stream bank on the west (approx. 60m wide × 4m deep).

2.1 PUBLISHED GEOLOGY

Regional geological maps indicate the site is underlain by Holocene River deposits, comprising Poorly consolidated alluvial gravel, sand and mud (GNS 2011) (Figure 1).

2.2 ACTIVE FAULTS

No active faults directly impacting the site have been identified in the New Zealand Active Faults Database (GNS Science, 2018).

2.3 MAPPED LIQUEFACTION SUSCEPTIBILITY

The proposed development is mapped as having “Medium” liquefaction vulnerability in accordance with the Hawke’s Bay Emergency Management Group hazard portal.

2.4 FLOODING RISK

The proposed subdivision is mapped as having low risk of flooding in accordance with the Hawke’s Bay Emergency Management Group hazard portal.

2.5 SLOPE STABILITY AND EROSION

The proposed development is located east of Karamu stream bank and it’s at risk of slope instability and erosion.

2.6 GEOHAZARDS

A summary of potential geological hazards present on site is in Table 2. The risk assessment based on a review of Hawkes Bay Hazard Portal (2022) and our geotechnical investigations.

TABLE 1: SUMMARY OF POTENTIAL NATURAL HAZARDS

Hazard	Significant Risk to Site
Active Faults	Low
Liquefaction Susceptibility	Moderate
Earthquake Amplification	Moderate to High
Slope Stability	Moderate
Flood Risk	Low
Tsunami Risk	Near & Distant Source Inundation Extent
Expansive Soil	Low
Coastal Hazard Zone	Low

3 SITE INVESTIGATION

Site testing has been undertaken with reference to relevant HDC and MBIE guidelines as in Section 1.2.

RDCL engineering geologists and technicians completed site investigations:

- Fourteen (14) Machine Augured boreholes (MA):
  - Excavated to between 0.9 m to 1.4m bgl.
- Two Hand Augers (2) Hand Augers tests:
  - Terminated between 1.0m to 3.0m bgl.
- Eight (8) Cone Penetrometer Tests (CPT):
  - Terminated between 8.2m and 9.6m bgl.
- One (1) Sonic Drilled Borehole (CPT):
  - Drilled to 15.45m bgl.

CPT, MA, HA and BH logs are attached as Appendix A.

3.1.1 HDC (2019) Guidelines for Liquefaction

Geotechnical investigations for this project have been undertaken in accordance with HDC (2019) guidelines for Building Consent. The site is located within "Medium" liquefaction susceptibility and HDC (2019) guidelines stipulate the necessity to perform CPT tests to 10m – 15m bgl.

- CPT testing was unable to reach 10.0m to 15.0m due to dense gravel layer ~ 8.5m to 9.0m bgl.
- A BH to15.5m bgl were utilized to assess liquefaction.

We believe the ground model obtained from completed ground testing is representative of the site conditions.

3.2 Subsurface Conditions

Results of subsurface investigation are shown in the Table 2.

Table 2 Ground Model

Depth From (m)	Depth To (m)	Material Description	Consistency / Density	SPT N Value
0	~ 0.2	Concrete Slab	NA	
0.2	Between 0.2 to 0.6	Basecorse gravel	Dense	
0.2 to 0.6	Between 0.3 – 0.8	Non engineered Fill and organic Silt	Stiff	3
0.3 – 0.8	3.0	Silty Sand and Sand with isolated clay lenses	Loose	7
From BH01				
3.0	5.0	Pumiceous Sand	Loose	18
5.0	6.6	Sandy Silt with some organics	Firm	7
6.6	8.1	Clay with trace of organics	Soft	2
8.1	10.0	Sand and sandy Gravel	Dense	50+
10.0	11.8	Clay	Firm - Stiff	9
11.8	15.45	sandy Gravel	Dense	50+

Engineering geological logging of materials recovered from test locations is according to New Zealand Geotechnical Society guidelines (NZGS, 2005).

3.2.1 Groundwater

Groundwater was not encountered in any of test locations. Groundwater is likely to be ~ 4.0m bgl according to water level in Karamu Stream. A ground water level of 2.5m adopted for liquefaction assessment.

4 GEOTECHNICAL ASSESSMENT (RISK BASED APPROACH)

A qualitative based risk assessment has been adopted for the proposed development to address:

- Stability of the Karamu stream bank; and
- Liquefaction induced land damage.

Inputs for the assessment include:

- A cross section developed during site mapping;
- Geotechnical materials and groundwater level as logged by engineering geologist onsite;
- Proximity of proposed platforms to the stream bank as supplied.

Analyses stream bank stability, potential effect on adjacent structures due to liquefaction and slope failure have been assessed using Limit equilibrium methods, and Finite Element Modelling.

4.1 NUMERICAL STABILITY AND LIQUEFACTION ASSESSMENT

The assessments are based on stability and liquefaction analysis undertaken on:

- The cross-section A-A' (Figure 01), using;
  - Slide 2 & RS2, program developed by Rocscience Inc.
- Eight (8) CPT tests, using;
  - CLiq, program developed by Geologismiki,
- One (1) sonic BH with SPT, using
  - LiqSVs, program developed by Geologismiki.

4.1.1 Basis of Assessment

Liquefaction assessment, slope stability and Finite element modelling for the site was based on CPT, MA and BH investigations and assessed using input parameters in accordance with MBIE Earthquake Geotechnical Engineering Practice in New Zealand. Module 1. Overview of guidelines (November 2021):

- Magnitude (M) = 6.4 (SLS), & 7.1 (ULS);
- Peak Ground Acceleration (PGA) = 0.12g (SLS) & 0.58g (ULS);
- Water table at 2.5 m bgl;
- A 50-year design life was assigned;
- The site is classified as site subsoil "Class D – Deep Soil Site"
- Importance Level category 2.

Assessment parameters and results of analyses are presented in Appendix B.

4.1.2 Geotechnical Parameters

Geotechnical parameters for stability analyses are in Appendix B.

The distribution of materials is based on geotechnical logs from One (1) sonic BH, Machine Augers and CPT tests.

4.2 Assessments

4.2.1 Slope Stability

The moderately steep (~ 3m high) west facing stream bank below the proposed development does not show any obvious indication of instability in the current state.

The slopes are likely to be affected with lateral spread during an ULS seismic event.

The slope has been assessed using numerical analyses carried out using industry accepted softwares Slide2 and RS2 on cross section developed from site mapping (Appendix B) and the ground model from testing.

4.2.2 Slope Stability Assessment

Slope stability was assessed using:

- Limit equilibrium methods to assess the stability of the stream bank under Static and Seismic (SLS & ULS) conditions; and
- Finite Element model developed on the most critical case (ULS event) to assess displacement.

Results indicate the slope is:

- Stable under static and SLS seismic conditions; and
- At risk of failure under ULS seismic event; with
- The soils up to ~5.5m bgl likely to experience lateral displacement.

Results from slope stability modelling are presented in Appendix B and summarised below.

Table 3– Critical Results – Section A -A'

Design Case	Factor of Safety	Overall Displacement (mm)
Static	> 2	Not Assessed
Seismic - SLS	> 1.7	Not Assessed
Seismic - ULS	≤ 0.7	400

4.2.3 Liquefaction Assessment

A liquefaction assessment was carried out using the results of CPT and Borehole (BH01) investigations and industry standard softwares CLiq And LiqSVs (Appendix B).

This assessment is based on the two limit states that are defined by AS/NZ 1170.0:2002; the serviceability limit state (SLS) and the ultimate limit state (ULS).

Liquefaction severity number (LSN) induced vertical settlement and lateral displacement results are summarized below.

Table 4 ULS: LSN, Estimated Free-Field Vertical Settlement & Lateral Displacement

CPT ID	Liquefaction Severity (LSN)	Vertical Settlement (mm)	Lateral displacement (mm)
CPT01_ULS	Minor surface expression	73	1177
CPT02_ULS		51	666
CPT03_ULS		50	466
CPT04_ULS	Little to no surface expression	37	443
CPT05_ULS		29	237
CPT06_ULS		30	383
CPT07_ULS		28	461
CPT08_ULS	Minor surface expression	63	771
BH01_ULS		67	450

Table 5 SLS: LSN, Estimated Free-Field Vertical Settlement & Lateral Displacement

CPT ID	Liquefaction Severity (LSN)	Vertical Settlement (mm)	Lateral displacement (mm)
CPT01_SLS	Little to no surface expression	3	11
CPT02_SLS		2	5
CPT03_SLS		1	3
CPT04_SLS		2	9
CPT05_SLS		0	3
CPT06_SLS		3	4
CPT07_SLS		1	3
CPT08_SLS		3	15
BH01_SLS		0	0

Estimated lateral displacement along the Karamu Stream were found to be ≥ 1.1 m.

4.3 SHALLOW BEARING CAPACITY

No DCP tests were completed at the time of investigation. Based on result of CPT and MA investigations, an Ultimate Bearing Capacity (UBC) is likely to be available at a level cleared of non-engineered fill ~ 0.8m bgl. This must be confirmed by site specific testing at building consent stage level.

5 ENGINEERING CONSIDERATIONS

5.1 Free of Significant Natural Hazard

The site has a risk of liquefaction induced lateral spread which is most evident at or near the crest of the slope to the Karamu Stream. Remedial options are available including commonly applied remedial measures are available to manage the hazard to acceptable levels and allow building (Section 5.2.1).

The site is otherwise considered free of geo-hazard and suitable for resource consent from a geotechnical perspective.

5.2 Suitability for development

Based on the results of these investigations and current topography of the site, we consider the site is suitable for the proposed development subject to the geotechnical recommendations presented in this report.

5.2.1 Summary of Measures

The proposed mitigation measures required based on this risk assessment is summarised in Table 6.

Table 6 Mitigation Measures

<table>
<tr><th>Risk</th><th>Level</th><th>Mitigation</th><th>Ground improvement</th></tr>
<tr><td>Liquefaction induced Lateral spread</td><td>High (1117mm_ULS)</td><td>Significant setback with TC3 compliant foundations</td><td rowspan="4">Construct an inground wall to address stability and lateral spread risk with TC2 compliant foundations</td></tr>
<tr><td>Liquefaction induced Settlement</td><td>Moderate
73mm _ULS</td><td>TC2 compliant Foundations</td></tr>
<tr><td>Slope Stability</td><td>High</td><td>Significant setback</td></tr>
<tr><td>Horizontal Displacement</td><td>High</td><td>Significant setback</td></tr>
</table>

5.2.2 Lateral Spread Governs

Lateral spread due to ULS earthquake govern geotechnical works for development of this site, with bearing and stream bank stability are nominally a secondary issue.

To mitigate against the effect of lateral spread:

- A 9.0m setback from crest of the stream bank (break in the slope) must be maintained; or
- Alternatively, an inground wall to reduce the potential for liquefaction and slope failure induced displacement.

Then:

- A compliant TC2 waffle slab can be constructed over a soil raft, being most likely:
  - A reinforced gravel raft; or
  - Cement stabilised crust.

5.3 Inground Wall

If the client wishes to build within the setback zone, to protect the proposed Structures against liquefaction induced damage and potential slope failures, a specifically engineered designed inground wall is required.

The inground wall must be specifically designed, as a guidance to help envisage the wall, following design is likely to be suitable, comprising of:

- Timber SED poles, nominally 300mm diameter, ~8m long;
- Driven on a closely spaced "dice-5" pattern, to
- Form an inground wall:
  - extending along the line of the Karamu Stream bank;
  - Situated a minimum 5m from the houses;
  - with sufficient overlap along the stream bank to protect the houses from damage.

The concept of the inground wall is to densify the ground sufficient that it will not liquefy. The non-liquefied wall then acts to restrict any liquefaction induced displacement, and to protect the houses.

5.1 CONFIRMATION OF DESIGN

Geotechnical design of foundations and mitigations should be confirmed based on building locations and in conjunction with architect and the client.

we suggest a design meeting is co-ordinated, with all parties and RDCL, so that the most suitable and cost-effective solution can be identified.

6 REFERENCES

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10. Stockwell, M. (1977). Determination of allowable bearing pressure under small structures. *New Zealand Engineering*, 32(6), pp.132-135.

7 LIMITATIONS

- This report has been prepared for the particular purpose outlined in the project scope and no responsibility is accepted for the use of any part in other contexts or for any other purpose.
- Ground conditions assessed in this report are inferred from published sources, site inspection and the investigation described. Variations from the interpreted conditions may occur, and special conditions relating to the site may not have been revealed by this investigation, and which are therefore not taken into account. No warranty is included either expressed or implied that the actual conditions will conform to the interpretation contained in this report.
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We trust this meets your current needs. Should you wish to discuss any aspect of the contents of this document please contact the undersigned on 06 877-1652.

Sincerely,

Prepared by:	Approved by:
B Bistouni	CA Wylie
MSc	MSc; MIPENZ, CPEng
Senior Engineering Geologist	Principal

FIGURE 1: INVESTIGATION LAYOUT

APPENDIX A– INVESTIGATION LOG OUTPUTS

APPENDIX B– ASSESSMENT LOG OUTPUTS